Chapter 4
Nearshore Breakwaters

4-1. Purpose

a. Scope. The purpose of this chapter is to provide design guidance on nearshore breakwaters and submerged sills. Their advantages and disadvantages are presented along with how they influence waves, what their effect is on the shoreline, and when they are viable options as effective shoreline stabilization methods. General information and a review of detached breakwater projects in the United States can be found in Dally and Pope (1986), Pope (1989), and Kraft and Herbich (1989).

b. Nearshore breakwaters. Offshore breakwaters are generally shore-parallel structures that effectively reduce the amount of wave energy reaching the protected stretch of shoreline. They can be built close to the shoreline they are intended to protect, in which case they are called nearshore breakwaters, or they can be built farther from shore. When used for beach stabilization, breakwaters function to reduce wave energy in their lee and thus reduce the sediment carrying capacity of the waves there. They can be designed to prevent the erosion of an existing beach or a beach fill, or to encourage natural sediment accumulation to form a new beach. Figure 4-1 depicts the basic characteristics of a single detached breakwater.

c. Submerged sills. Submerged sills are also generally shore-parallel and built nearshore. Their purpose is to retard offshore sand movement by introducing a structural barrier. The shore-parallel sill interrupts normal offshore sediment movement caused by storm waves; however, it may also interrupt the onshore movement caused by "beach building" waves. The sill introduces a discontinuity into the beach profile so that the beach behind it is at a higher elevation (and thus wider) than adjacent beaches. The beach is thus "perched" above the surrounding beaches. Figure 4-2 depicts the basic perched beach concept.

d. Difference. A distinction between nearshore breakwaters and submerged sills can be made by noting their effects on waves and sediment transport. Breakwaters act to reduce waves; submerged sills act as barriers to shore-normal sediment motion. The primary characteristics that determine how a structure is classified is the structure's crest elevation. Breakwaters have crest elevations high enough to significantly reduce the height of waves transmitted over them. (Waves in the lee of a breakwater can also result from diffraction around the breakwater's end and transmission through the breakwater.) The effect of submerged sills on waves is relatively small because their crest elevation is at or below the water level. Their crest may be exposed at low tide; however, at most stages of tide, they are submerged. While sills may trip some large waves into breaking, they simply provide a barrier to onshore and offshore sediment movement at one point on the beach profile.

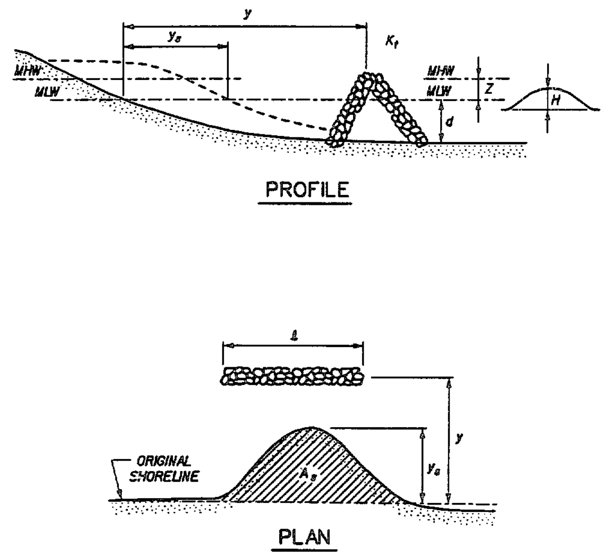


Figure 4-1. Schematic of a single detached breakwater

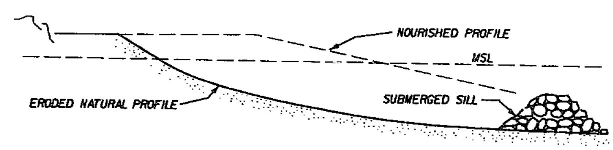


Figure 4-2. Submerged sill and perched beach concept

4-2. Design Objectives

The primary design objective of a nearshore breakwater or submerged sill system is to increase the longevity of a beach fill, provide a wide beach for recreation, and/or afford protection to upland areas from waves and flooding. In addition, adverse effects, usually erosion along downdrift beaches due to a breakwater's halting or reducing the normal longshore transport, should be minimized.

a. Advantages of breakwaters. Nearshore breakwaters offer several advantages over other beach stabilization structures. First, if properly designed, they effectively control erosion and retain sand on a beach. Second, they reduce the opportunity for rip currents to form and thus reduce offshore sediment losses. Third, they reduce the steepness of waves in their lee and encourage landward sand transport. Fourth, they reduce wave heights along a beach.

b. Disadvantages of breakwaters. Breakwaters also have several disadvantages. There is only a limited amount of US prototype experience with nearshore breakwaters for shoreline protection, although Japan and several Mediterranean countries have had extensive experience with these structures. In addition, design guidance, especially in the planning stages of a project, is somewhat limited. Because they are located offshore, nearshore breakwaters can be expensive to build and may require the use of temporary trestles or barge-mounted construction equipment. Similarly, they may be expensive to maintain because of their offshore location. The gaps between a series of breakwaters can channel flow and sediment offshore if water levels behind the breakwaters build up as a result of wave overtopping. Relatively high offshore velocities through these gaps can scour the bottom unless riprap armoring is provided. Breakwaters can also be a total barrier to longshore sand transport unless care is taken to ensure that some wave energy is available behind them to transport sand. Thus, they can totally halt the flow of sand to downdrift beaches and cause erosion there. Breakwaters can also be hazardous to bathers and swimmers if they climb on the structures or get caught in offshore flows. They can also reduce the potential for recreational surfing in the project area.

c. Beach planform. A primary consideration in the design of a nearshore breakwater for beach stabilization purposes is the desired planform and beach width behind the breakwater. Basically, three different types of shorelines can develop behind a breakwater or a system of breakwaters (Figure 4-3). If the breakwater is close to shore, long with respect to the length of the incident waves, and/or sufficiently intransmissible to the average waves, sand will continue to accumulate behind the breakwater until a tombolo forms; that is, the shoreline continues to build seaward until it connects with the breakwater. If a tombolo forms, longshore transport is stopped until the entire updrift beach fills seaward to the breakwater and sand can move around its seaward side. The breakwater-tombolo combination functions much like a T-groin. If the breakwater is far from shore, short with respect to the length of the incident waves, and/or relatively transmissible, the shoreline will build seaward, but is prevented by wave action and longshore currents from connecting with the breakwater. The shoreline bulge that forms is termed a "salient." If a salient forms, longshore sand transport rates are reduced; however, transport is not completely stopped. The third beach type is termed limited shoreline response in which little beach planform sinuosity is experienced, possibly due to a lack of adequate sediment supply. The final shoreline configuration and its location depend on the geometry of the breakwater system, the wave environment, the longshore transport environment, and the amount of available sand. The variability of wave height, period, and direction coupled with the geometry of the breakwater system are all important in determining the final equilibrium planform of the beach.

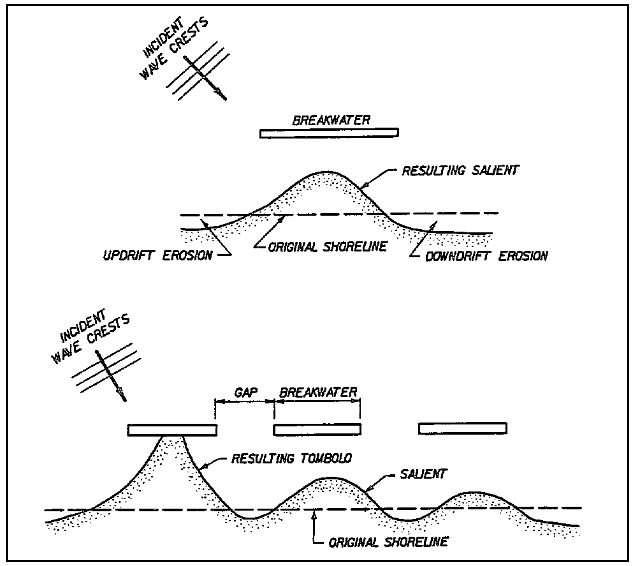


Figure 4-3. types of shoreline changes associated with single and multiple breakwaters and definition of terminology

d. Types of nearshore breakwaters. Nearshore breakwaters can be classified by type of construction and by their planform geometry and crest elevation. There are four basic forms of nearshore breakwaters for shore stabilization. They are a single detached breakwater, a multiple detached breakwater system, artificial headlands, and a submerged sill structure intended to form a perched beach.

(1) A single detached breakwater generally has a limited range of influence and thus protects only a local

reach of shoreline. However, a significant distance of shoreline updrift and downdrift of the breakwater can be affected if a tombolo forms at the structure. Critical design dimensions for a single breakwater are its length, distance offshore, and crest elevation. These dimensions determine whether a tombolo will form and whether the longshore transport rates will prevail following construction.

(2) A multiple breakwater system can be constructed to protect a longer stretch of shoreline. If properly designed, a multiple breakwater system can continue to maintain a reduced rate of longshore transport past a project area, thus minimizing downdrift erosion. Critical design dimensions for multiple breakwater systems are the length of the individual breakwater elements, distance offshore, distance between breakwaters (gap width), and crest elevations. The shape of the resulting shoreline and the amount of transport through the project area depend on these parameters.

(3) Offshore breakwaters have been used as artificial headlands in an attempt to create stable beaches landward of the gaps between the structures (Silvester 1970, 1976; Chew et al. 1974; USAED, Buffalo 1986; Pope 1989; Hardaway and Gunn 1991). A definition sketch for an artificial headland breakwater system is provided in Figure 4-4. As opposed to detached breakwaters where tombolo formation is often discouraged, artificial headland systems are designed to form a tombolo. Artificial headland design parameters include the approach direction of dominant wave energy, length of individual headlands, spacing and location, crest elevation and width of the headlands, and artificial nourishment (Bishop 1982; USAED, Buffalo 1986).

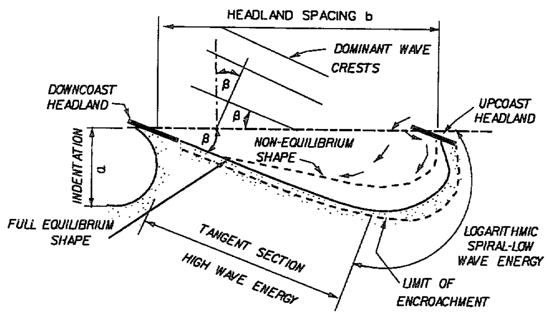


Figure 4-4. Definition sketch of an artificial headland breakwater system and beach planform

(4) Submerged sills can be classified as nearshore breakwaters with crest elevations that are below the mean tide level. They can be built with or without shore-return structures to connect the offshore sill with the shoreline. The shore-return structures and sill hold a beach fill within a boxlike compartment, with the shore returns functioning like groins. Little to no documented experience exists for submerged sills and perched beaches along the exposed ocean coastlines of the United States. There has been some limited experience with perched beach sills in sheltered waters (Dunham, et al., 1982; Douglass and Weggel 1987). This experience suggests that submerged sills slow offshore losses from an area, but that periodic nourishment of the compartment is still necessary to maintain a wide beach. Important design parameters include the sill length, distance offshore, crest elevation, and whether or not to include shore-return structures in the design.

e. Structural effects and design parameters.

(1) Length of shoreline to be protected.

(a) The length of shoreline protected by a single breakwater (and also the downdrift length of shoreline that might be adversely affected by a single breakwater) depends on whether or not the breakwater forms a tombolo. If a tombolo forms in a continuous littoral system, the effect of the breakwater will be to accumulate sand along updrift beaches and to starve downdrift beaches. If located in an area where the net longshore transport is close to zero, the breakwater's range of influence will be limited to the general vicinity of the structure, and the effects may not extend very far updrift or downdrift. If a longer portion of the shoreline must be protected, a system of several breakwaters spaced along the shoreline with gaps between them must be constructed. Building a single long breakwater will not achieve the same result, but will result in the formation of a single tombolo or of two tombolos, one extending seaward from shore to each end of the breakwater. The resulting lagoon enclosed by the breakwater and tombolos is usually undesirable. A multiple breakwater system with gaps also reduces the amount of material needed for construction. In most cases, when there is a net direction of longshore transport, tombolos are unwanted because of the downdrift erosion caused by totally interrupting longshore transport. Generally, a system of multiple nearshore breakwaters is needed to protect a long reach of shoreline while still maintaining some longshore transport to minimize erosion along downdrift beaches.

(b) Wave heights behind a nearshore breakwater can be significantly reduced. Waves in the lee of a breakwater get there by transmission through the structure if it is permeable, regeneration in the lee of the structure by overtopping waves, and diffraction around the ends of the breakwater. If the breakwater's crest elevation is high and it is impermeable, diffraction is the primary source of wave energy in the shadow zone. Wave diffraction is discussed in the SPM (1984, Chapter 3, Section IV). For a detached breakwater, waves propagate around each end of the breakwater and interact in its lee. Wave heights become smaller farther behind the breakwater. If the incident waves are nearly monochromatic, they interact constructively or destructively behind the breakwater, depending on whether the crest and trough of the waves coming around each end are in or out of phase with each other. Thus, there are regions behind the breakwater where monochromatic waves nearly cancel each other and other areas where they reinforce each other. If a range of wave periods is present, as it often is in the prevailing wave spectrum, a more uniform distribution of wave heights prevails in the breakwater's lee. As the direction of incoming waves changes, the salient in the sheltered area behind the breakwater responds by repositioning itself in the region to the structure's lee. A diffraction analysis should be used to determine the approximate shoreline configuration behind a breakwater. Studies indicate that if the isolines of the K' = 0.3 diffraction coefficients are constructed from each end of the breakwater for a range of incident wave directions and they intersect seaward of the postproject shoreline, a tombolo will not form (Figure 4-5) (see Walker et al. 1980). More simply, this is ensured if the breakwater lies more than one half the breakwater's length seaward of the postproject shoreline, i.e. after placement of beach fill if that is part of the project. Waves coming around each end of the breakwater meet each other before the undiffracted incident wave (outside of the breakwater's shadow) reaches the shoreline. The postproject shoreline can be estimated by drawing the pattern of the diffracted wave crests behind the breakwater and smoothing the crest pattern to balance the amount of sediment available.

(2) Types of construction.

(a) Most US and foreign nearshore breakwaters built for shore protection have been rubble-mound structures. Several structures have been built of steel sheet-pile cells in the Great Lakes; however, these structures were not intended to function as shore protection, but rather to protect a harbor entrance from waves (for example, Vermilion Harbor, Ohio). Their effect on adjacent shorelines, however, has been similar to that of shore

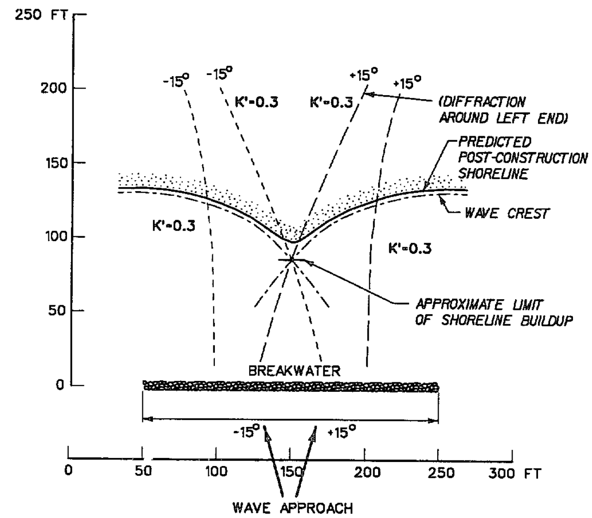


Figure 4-5. Estimate of post project shoreline behind a detached nearshore breakwater, isolines of diffraction coefficient, K' = 0.3

stabilization breakwaters. Rubble-mound construction of nearshore breakwaters is advantageous since rubble-mound structures dissipate more incident wave energy and are relatively easy to construct in the nearshore zone. Several patented shore protection devices that function like nearshore breakwaters have been built, mostly in sheltered waters. Some of these have been tested under the Shoreline Erosion Control Demonstration Act (Dunham et al. 1982). Several have been built of precast reinforced concrete units; others have been built of concrete blocks and sand-filled geotextile tubes and bags. Refer to EM 1110-2-2904 for further guidance on the design of rubble-mound and other type structures.

(b) Submerged sills of various types have been built in sheltered waters. Sand-filled bags, timber sheet piles, and sand-filled precast concrete boxes have been used for sill construction. There does not appear to be a discernible difference in functional performance between the various types of sills; however, a sand-tight rubble-mound sill is recommended for perched beaches because of its ability to dissipate wave energy.

(3) Crest elevation.

(a) Crest elevation determines the amount of wave energy transmitted over the top of a nearshore breakwater or submerged sill. High crest elevations preclude overtopping by all but the highest waves whereas low crest elevations allow frequent overtopping. Generally, low crests allow more wave energy to penetrate into the lee of the breakwater. Occasional overtopping of a nearshore breakwater by storm waves can prevent tombolo formation or remove a tombolo once it has formed. For an artificial headland system, the amount of overtopping should be minimized to encourage tombolo formation. Wave transmission by overtopping is discussed in the SPM (1984, Chapter 7, Section II). Prediction of irregular wave overtopping of structures is discussed in Ahrens (1977).

(b) For a submerged sill, crest elevation determines the elevation and spatial extent of the perched beach that can be maintained behind the sill. Higher sills also have more effect on incident waves. While the primary purpose of a submerged sill is to retain sand, it also triggers breaking by some waves and reduces wave energy levels on the perched beach. As the sill elevation is increased, it begins to function more like an offshore breakwater; that is, its effect on waves increases.

(4) Circulation and modification of currents.

(a) Construction of offshore breakwaters and sills will result in significant changes in the nearshore current system. On a natural beach, shore-parallel longshore currents are generated by waves approaching the shoreline at an angle. If breakwaters are built, the driving force for the currents is intercepted by the breakwater along part of the shoreline. The prevailing longshore current, unless maintained by its inertia, will slow or stop when it moves into the sheltered area behind the breakwaters. The sand carrying capacity of the current and the wave agitation that suspends sediment so it can be carried by the current are reduced. A breakwater's length and distance from shore are critical in determining its effect on longshore currents and sediment transport. A long breakwater will cause the longshore current to slow and spread laterally and will shelter a long reach of shoreline from wave agitation.

(b) If the breakwater crest elevation is low enough to allow overtopping, water carried over the breakwater will raise the water level behind it and cause flow around the breakwater. In multiple breakwater systems, overtopping causes a net seaward flow of water through the gaps. Seelig and Walton (l980) present a method for estimating the strength of the seaward flowing currents. Return currents can be reduced by raising the breakwater crest elevation, enlarging the gaps between segments, or increasing structure permeability. For permeable breakwaters, some flow is also carried seaward through the breakwater itself.

(5) Effect on wave environment.

(a) Breakwaters reduce the amount of wave energy reaching the shoreline. Wave heights in the lee of a breakwater are much lower than they are in the exposed area seaward of the breakwater. Waves in the lee of a breakwater are determined by three processes: diffraction around the breakwater ends, wave transmission by overtopping, and wave transmission through the structure. Local diffracted wave heights are determined primarily by their exposure and distance from the breakwater's ends or, in the case of a multiple breakwater system, by their location relative to the breakwater gaps (see SPM (1984), Chapter 2, Section IV). Wave heights due to overtopping are determined by the breakwater crest elevation. Wave transmission through a breakwater is determined by its permeability (SPM (1984) Chapter 7, Section II; Madsen and White 1976; Seelig 1979, l980). The Automated Coastal Engineering System (Leenknecht et al. 1990) provides an application to determine wave transmission coefficients and transmitted wave heights for permeable breakwaters with crest elevations at or above the still-water level. This application can be used with breakwaters armored with stone or artificial armor units.

(b) Wave conditions seaward of a breakwater are determined by its reflection characteristics. Reflected waves interact with incident waves to cause a partial standing wave pattern seaward of a breakwater. Agitation of bottom sediments by standing waves can cause scour and undermining seaward of the breakwater and contribute to other foundation problems. Reflection characteristics are in turn determined by breakwater permeability, crest elevation, and type of construction. Permeable, low-crested, rubble-mound breakwaters are the least reflective structures; however, they can allow significant amounts of energy to propagate through them. Rubble-mound structures dissipate wave energy by inducing fluid turbulence in their interstices.

(6) Effect on longshore transport.

(a) Nearshore breakwaters reduce longshore transport rates by sheltering a reach of shoreline from waves. Much like a groin, the breakwater forms a partial or total barrier to longshore transport. The reduction in transport capacity is determined by both a reduction in wave height in the breakwater's lee and by redirection of wave crests by diffraction around the breakwater's ends. Long single breakwaters or closely spaced multiple breakwaters can form a near complete barrier to longshore transport. If a tombolo forms, transport is almost totally interrupted, with the exception of transport seaward of the breakwater. For breakwaters where only salients develop, longshore transport rates can be adjusted to meet desired design objectives. Sediment budget analyses should be made to determine the effect of a transport rate reduction on both updrift and downdrift beaches under postproject conditions. Adjusting the length, distance offshore, and crest elevation of a single breakwater will vary the resulting longshore transport rate. For multiple breakwater systems, gap width may also be modified. A fixed-bed physical model with a sediment simulant tracer can be useful in estimating and comparing pre- and postproject transport rates for various cases.

(b) In general, the effect of submerged sills on longshore sediment transport is relatively small. Since there is a small reduction in incident wave energy, there will be some reduction in transport rates within a perched beach. In cases where the breaking wave angle is relatively small, there may be a more significant effect on longshore transport. If shore-return structures are included in a perched beach design, they will affect longshore transport similar to low groins, and the rate of longshore transport into and out of the perched beach area will be reduced.

(7) Effect on onshore-offshore transport.

(a) Nearshore breakwaters can reduce offshore sand transport. Wave heights in a breakwater's lee are reduced, and their direction is changed. Lower wave heights result in waves with a lower wave steepness (wave-height-to-wavelength ratio) and are therefore more likely to transport sand onshore than offshore. For multiple breakwater systems, offshore sand losses are reduced; however, overtopping can result in a net seaward flow of water and sand through the gaps between breakwater segments. These currents usually occur when the structure is nearly impermeable and low crested so that the water transmitted by overtopping can return only through the gaps or around the ends of the structure. The breakwater can also reduce onshore sediment movement. Following breakwater construction, a new equilibrium between onshore and offshore transport will be established.

(b) Submerged sills are intended to reduce the rate of offshore sand transport. They establish a location on the beach profile across which both offshore and onshore transport is much reduced from what it would be across a normal profile. While the sill is intended to reduce offshore losses during storm wave conditions, it also reduces onshore movement during beach-building, low-steepness wave conditions. The sill's net effect on onshore-offshore transport processes has not been quantitatively established; consequently, it is not known whether the sill's overall effect is beneficial or detrimental. A laboratory study by Sorensen and Beil (1988) investigated the response of a perched beach profile to storm wave attack.

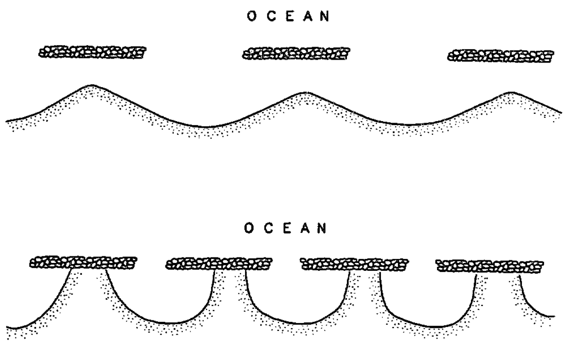
f. Design to meet functional objectives.

(1) A single detached breakwater or multiple breakwater system generally has as its primary objectives to increase the life of a beach fill, provide a wide beach for recreation, and/or protect upland development. To establish and protect a relatively short reach of shoreline (on the order of several hundreds of feet), a single breakwater can provide the needed sheltering. If a tombolo is allowed to form acting as a littoral barrier, a single breakwater's effects can extend a great distance upcoast and downcoast.

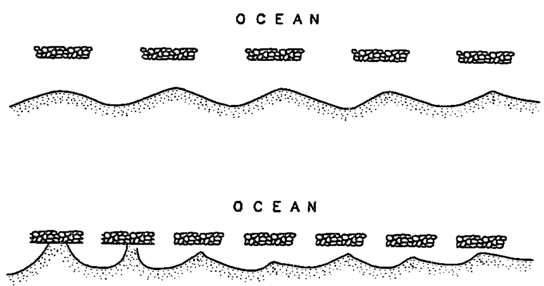
(2) If located in an area where the net transport is almost zero, but where the gross transport is not zero, the breakwater's major effects will be limited to the general vicinity of the breakwater itself. Minor effects, however, can extend to significant distances. The time period for the effects of a breakwater to be observed along updrift and downdrift beaches depends on both the net and gross transport rates. For large transport rates, the effects are felt quickly; for low rates, the effects may take years to appear.

(3) If a significant length of shoreline must be protected, a multiple breakwater system should be considered. The number of breakwaters, their size, and the gap width between them depend on the wave environment and the desired shape of the shoreline behind them. A few long, widely spaced breakwaters will result in a sinuous shoreline with large amplitude salients and a spatial periodicity equal to the spacing of the breakwaters; that is, there will be a large salient behind each breakwater (Figure 4-6a). Numerous, more closely spaced breakwaters will also result in a sinuous shoreline, but with more closely spaced, smaller salients (Figure 4-6b).

(4) Wide gaps permit more wave energy to penetrate into the area behind the breakwaters, thus maintaining



a. With a few relatively long, widely spaced segments



b. With more numerous, shorter, closely spaced segments

Figure 4-6. Multiple breakwater system

some level of longshore sand transport. The ratio of gap width to the sum of breakwater length and gap width for various prototype projects (the fraction of the shoreline directly open to waves through the gaps, termed the "exposure ratio") ranges from about 0.25 to 0.66. Table 4-l provides examples of various prototype projects and their associated exposure ratios. Projects like Presque Isle, PA, and East Harbor State Park, OH, where the purpose is to contain a beach fill within fixed project boundaries have larger exposure ratios. Comparatively, the exposure ratio at Lakeview Park, Lorain, OH, is 0.36, and at Winthrop Beach, MA, where the gaps were included to allow for small craft navigation, the ratio is 0.25.

(5) The postproject shoreline configuration can be determined from diffraction analyses using a range of wave conditions characteristic of the site. The design process is one of trial and error. A trial breakwater configuration is assumed based on past experience at existing breakwater systems. Then the trial configuration is evaluated to determine if it will satisfy the project's objectives. Its effect on the shoreline and on the overall sediment budget of the project area and adjacent beaches is evaluated. The trial configuration is adjusted and the modified project's effects evaluated. Evaluation tools for proposed breakwater configurations include the interpretation of diffraction analyses, overtopping analyses, and other manual computations; physical model tests of the proposed project configuration; and numerical computer simulations of shoreline evolution. Because of the limited experience with prototype detached breakwaters in the United States, a great deal of engineering judgment and comparison with the few existing breakwater projects is necessary.

(6) Dimensional analysis can provide some insight into the design of single and multiple nearshore breakwater systems. A more detailed section on dimensional analysis of detached breakwaters and an example application can be found in Appendix D.

g. Empirical relationships for breakwater design.

(1) Summary of relationships.

(a) The functional design and prediction of beach response to single and segmented detached breakwaters systems have been the subject of numerous papers and reports (SPM 1984; Gourlay 1981; Ahrens and Cox 1990; Dally and Pope 1986; Suh and Dalrymple 1987; Nir 1982; Noble 1978; Inman and Frautschy 1966). A number of these references have been reviewed in Rosati (1990) and are summarized in Table 4-2. A design procedure developed by the Japanese Ministry of Construction (JMC) (1986) has been summarized by Rosati and Truitt (1990). Most references present morphological information on when tombolos will form and when minimal beach response to breakwater construction can be expected. These conditions are usually specified in terms of the dimensionless breakwater length, ℓ/y, where y is the distance from the average shoreline; or the breakwater length-to-wavelength ratio, ℓ/gT^2, where g is the acceleration of gravity and T is the wave period. The other dimensionless parameters given in Appendix D are also important and play a role in determining how the shoreline responds to nearshore breakwater construction.

(b) Conditions for tombolo formation cited by various investigators are given in Table 4-3. The conditions for salient development are given in Table 4-4, and the

Table 4-1
"Exposure Ratios" for Various Prototype Multiple Breakwater Projects*

Project	Exposure Ratio
Winthrop Beach, MA	0.25
Lakeview Park, Lorain, OH	0.36
Castlewood Park, Colonial Beach, VA	0.31 to 0.38
Central Beach, Colonial Beach, VA	0.39 to 0.45
East Harbor State Park, OH	0.56
Presque Isle, Erie, PA	
(experimental prototype)	0.56 to 0.66
(hydraulic model)	0.60

* The "exposure ratio" is defined as the ratio of gap width to the sum of the breakwater length and gap width. It is the fraction of shoreline directly exposed to waves and is equal to the fraction of incident wave energy reaching the shoreline through the gaps. A "sheltering ratio" that is the fraction of incident wave energy intercepted by the breakwaters and kept from the shoreline can also be defined. It is equal to 1 minus the "exposure ratio."

conditions for limited shoreline response are given in Table 4-5.

(c) Other empirical relationships for other variables have been proposed. For example, Suh and Dalrymple (1987) propose the following relationships for the length of the salient behind a single breakwater:

$$\frac{y_s}{\ell} = 0.156 \text{ for } \frac{y_b}{y} < 0.5 \qquad (4\text{-}1)$$

$$\frac{y_s}{\ell} = 0.317 \text{ for } 0.5 < \frac{y_b}{y} < 1.0 \qquad (4\text{-}2)$$

$$\frac{y_s}{\ell} = 0.377 \text{ for } \frac{y_b}{y} > 1.0 \qquad (4\text{-}3)$$

where y_b is the distance from shore to the breaker line and y_s is the distance to the salient end from the average shoreline. Behind multiple breakwaters, Suh and Dalrymple (1987) propose,

$$y_s = 14.8\ y\ \frac{by}{\ell^2} \exp\left[-2.83\sqrt{(by/\ell^2)}\right] \qquad (4\text{-}4)$$

for the length of the salient, where b is the gap width.

(2) JMC method.

(a) Rosati and Truitt (1990) have summarized a procedure developed by the JMC for the design of a system of nearshore breakwaters. The procedure, developed from observations of the performance of a number of Japanese prototype breakwaters, results in a system of relatively short breakwaters located close to shore. Beach nourishment was not included in most of the prototype projects on which the procedure is based. Five different shoreline types were investigated. Type A is for shallow offshore areas, small wave heights, beach slopes of about 1:30, and fine sand. Type B is for beaches with well-developed offshore bars, gentle slopes (1:30), moderate wave heights, and mostly shore-normal incident waves. Type C is for relatively steep slopes (1:15), no offshore bar, moderate wave heights, and beaches of coarse sand and pebbles. Type D is for more steeply sloping beaches (1:3 to 1:10), moderate wave heights, and pebbles. Type E is similar to Type C but with an offshore bar. Detailed descriptions of the coasts for which the procedures were developed are given in Rosati and Truitt (1990). Sufficient data were available to develop detailed design procedures for Type B and C coastlines.

(b) The design wave used in the procedure is the average deepwater height of the five highest "nonstorm" waves occurring in a year, H_{o5}, and the wave period associated with that wave height, T_5. There is currently no simple way to relate this H_{o5} wave to other characteristic waves at a site such as the mean wave height or some other wave from the wave height distribution with a specified return period. H_{o5} is certainly less than the 1-year wave height (the wave height equaled or exceeded at least once in each year) but higher than the average daily wave height.

(c) After selecting the length of the shoreline reach to be protected and the desired shoreline advancement (salient length, y_s), the breaking water depth, d_b, of the H_{o5} wave is calculated using Figure 4-7 with the deepwater values of H_{o5} and L_{o5} (the deepwater wavelength associated with T_5). Calculate the ratio d'/d_b where d' is the water depth at the offshore breakwater estimated using $d' = (d_b + y \tan ß)/2$ where tan ß is the bottom slope. With the ratio d'/d_b, the salient area ratio (SAR) can be found from Figure 4-8. The SAR is given by,

Table 4-2
Summary of Empirical Relationships for Breakwater Design

Reference	Comment
Inman and Frautschy (1966)	Predicts accretion condition; based on beach response at Venice in Santa Monica, CA
Toyoshima (1972, 1974)	Recommends design guidance based on prototype performance of 86 breakwater systems along the Japanese coast
Noble (1978)	Predicts coastal impact of structures in terms of offshore distance and length; based on California prototype breakwaters
Walker, Clark, and Pope (1980)	Discusses method used to design the Lakeview Park, Lorain, OH, segmented system for salient formation; develops the Diffraction Energy Method based on diffraction coefficient isolines for representative waves from predominant directions
Gourlay (1981)	Predicts beach response; based on physical model and field observations
Nir (1982)	Predicts accretion condition; based on performance of 12 Israeli breakwaters
Rosen and Vadja (1982)	Graphically presents relationships to predict equilibrium salient and tombolo size; based on physical model/prototype data
Hallermeier (1983)	Develops relationships for depth limit of sediment transport and prevention of tombolo formation; based on field/laboratory data
Noda (1984)	Evaluates physical parameters controlling development of tombolos/salients, especially due to on-offshore transport; based on laboratory experiments
SPM (1984)	Presents limits of tombolo formation from structure length and distance offshore; based on the pattern of diffracting wave crests in the lee of a breakwater
Dally and Pope (1986)	Recommends limits of structure-distance ratio based on type of shoreline advance desired and length of beach to be protected
Harris and Herbich (1986)	Presents relationship for average quantity of sand deposited in lee and gap areas; based on laboratory tests
JMC (1986); also Rosati and Truitt (1990)	Develops step-by-step iterative procedure, providing specific guidelines towards final breakwater design; tends to result in tombolo formation
Pope and Dean (1986)	Presents bounds of observed beach response based on prototype performance; beach response given as a function of segment length-to-gap ratio and effective distance offshore-to-depth at structure ratio; provides beach response index classification scheme
Seiji, Uda, and Tanaka (1987)	Predicts gap erosion; based on performance of 1,500 Japanese breakwaters
Sonu and Warwar (1987)	Presents relationship for tombolo growth at the Santa Monica, CA, breakwater
Suh and Dalrymple (1987)	Gives relationship for salient length given structure length and surf zone location for single breakwater; based on laboratory tests
Berenguer and Enriquez (1988)	Presents various relationships for pocket beaches including gap erosion and maximum stable surface area (i.e., beach fill); based on projects along the Spanish coast
Ahrens and Cox (1990)	Uses Pope and Dean (1986) to develop a relationship for expected morphological response as function of segment-to-gap ratio
Ahrens (unpublished)	Extends results of Berenguer and Enriquez (1988)

Table 4-3
Conditions for the Formation of Tombolos

Condition	Comments	Reference
$\ell/y > 2.0$		SPM (1984)
$\ell/y > 2.0$	Double tombolo	Gourlay (1981)
$\ell/y > 0.67$ to 1.0	Tombolo (shallow water)	Gourlay (1981)
$\ell/y > 2.5$	Periodic tombolo	Ahrens and Cox (1990)
$\ell/y > 1.5$ to 2.0	Tombolo	Dally and Pope (1986)
$\ell/y > 1.5$	Tombolo (multiple breakwaters)	Dally and Pope (1986)
$\ell/y > 1.0$	Tombolo (single breakwater)	Suh and Dalrymple (1987)
$\ell/y > 2\ b/\ell$	Tombolo (multiple breakwaters)	Suh and Dalrymple (1987)

Table 4-4
Conditions for the Formation of Salients

Condition	Comments	Reference
$\ell/y < 1.0$	No tombolo	SPM (1984)
$\ell/y < 0.4$ to 0.5	Salient	Gourlay (1981)
$\ell/y = 0.5$ to 0.67	Salient	Dally and Pope (1986)
$\ell/y < 1.0$	No tombolo (single breakwater)	Suh and Dalrymple (1987)
$\ell/y < 2\ b/\ell$	No tombolo (multiple breakwaters)	Suh and Dalrymple (1987)
$\ell/y < 1.5$	Well-developed salient	Ahrens and Cox (1990)
$\ell/y < 0.8$ to 1.5	Subdued salient	Ahrens and Cox (1990)

Table 4-5
Conditions for Minimal Shoreline Response

Condition	Comments	Reference
$\ell/y \le 0.17$ to 0.33	No response	Inman and Frautschy (1978)
$\ell/y \le 0.27$	No sinuosity	Ahrens and Cox (1990)
$\ell/y \le 0.5$	No deposition	Nir (1982)
$\ell/y \le 0.125$	Uniform protection	Dally and Pope (1986)
$\ell/y \le 0.17$	Minimal impact	Noble (1978)

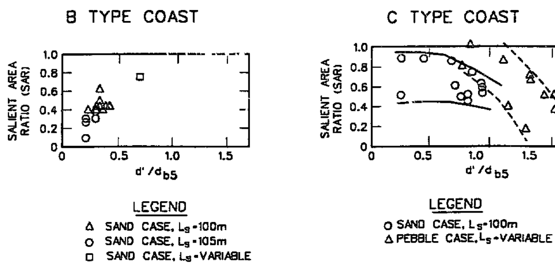


Figure 4-7. Deepwater wave steepness as a function of nearshore steepness for various beach slopes (Goda 1970)

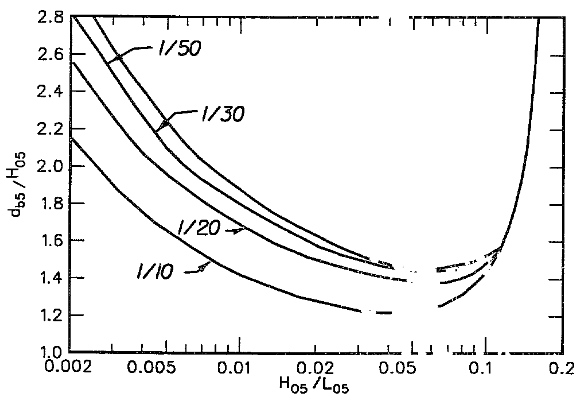


Figure 4-8. Salient area ration as a function of relative water depth, Type B shoreline and Type C shoreline

$$SAR = \frac{\frac{1}{2}(\ell_c y_s)}{y\ \ell} \tag{4-5}$$

in which ℓ_c is the salient length in the longshore direction measured at the original shoreline.

(d) The first approximation of the structure's distance offshore is given by $y' = d'/\tan\beta$. The first approximation of the salient extension is then given by $y_s' = SAR\ y'$. If this value of y_s' is approximately equal to the value of y_s' originally assumed, the value is adopted. If there is a significant difference, a new estimate of y_s' is made, and the above procedures are repeated until the two values are approximately equal.

(e) The range of structure lengths as a function of nearshore wavelength for beach Type B is given by,

$$1.8\ L_{o5} < \ell < 3.0\ L_{o5} \tag{4-6}$$

and for beach Type C by,

$$1.4\ L_{o5} < \ell < 2.3\ L_{o5} \tag{4-7}$$

The range of structure lengths as a function of offshore distance for beach Type B is given by,

$$0.8\ y < \ell < 2.5\ y \tag{4-8}$$

and for beach Type C by,

$$1.0\ y < \ell < 3.5\ y \tag{4-9}$$

Applying Equations 4-6 through 4-9 gives two ranges for the breakwater length. The breakwater length adopted is the average of the highest minimum and the lowest maximum of the two ranges.

(f) If the length of the shoreline to be protected exceeds twice the breakwater length, the gap width can be selected by using the following ranges of gap width that are valid for both Type B and Type C beaches,

$$0.7\ y < b < 1.8\ y \tag{4-10}$$

and,

$$0.5\ L_{o5} < b < 1.0\ L_{o5} < \tag{4-11}$$

As above, the gap width is selected as the average of the highest minimum and the lowest maximum of the two ranges.

(g) The calculated breakwater length, gap width, distance from shore, and SAR can then be used to develop a final breakwater system design subject to subsequent evaluation using analytical tools such as computer simulations, etc.

h. Artificial headlands.

(1) Artificial headlands or headland breakwaters are constructed either on or very near the original shoreline and are within the average surf zone (Pope 1989). They are designed to form a tombolo and function as a total block to the inshore littoral transport. Beach fill is usually incorporated into the project design, since these structures are not very efficient in trapping the regional longshore transport. Downdrift effects with headland breakwaters can be significant; therefore, they should be used in areas where there is minimum net littoral transport and the downdrift areas are not considered sensitive.

(2) A definition sketch of an artificial headland breakwater was provided in Figure 4-4, with the relationship between the variables, and thus the spacing and location of the breakwaters, presented in Figure 4-9 (Silvester et al. 1980; USAED, Buffalo 1986). The relationship between the spacing and indentation and the angle was derived from measurements of natural headland embayments known to be in equilibrium. When in equilibrium, a bay will experience no littoral drift movement since the predominant waves arrive normal to the beach at all points around the periphery (Silvester 1976). With sediment supplied, the shoreline will continue to be seaward of the static equilibrium position obtained using Figures 4-4 and 4-9, and longshore transport will continue to be bypassed downdrift.

(3) Most beaches associated with headlands assume a shape related to the predominant wave approach: a curved upcoast section representing a logarithmic spiral and a long and straight downcoast section (Chew et al. 1974). The logarithmic spiral shape of the beaches associated with headlands has been investigated extensively (Silvester 1970, 1974, 1976; Chew et al. 1974; Rea and Komar 1975; Silvester et al. 1980; Everts 1983; Berenguer and Enriquez 1988; Hsu et al. 1989).

(4) At artificial headland sites subject to bidirectional wave attack, the artificial headlands may have to be shore-connected with groins to prevent breaching. Alternatively, the breakwater length can be increased (USAED, Buffalo 1986).

i. Perched beaches.

(1) Perched beaches have not been studied extensively, and very few have been built; consequently, there is little information on which to base a design.

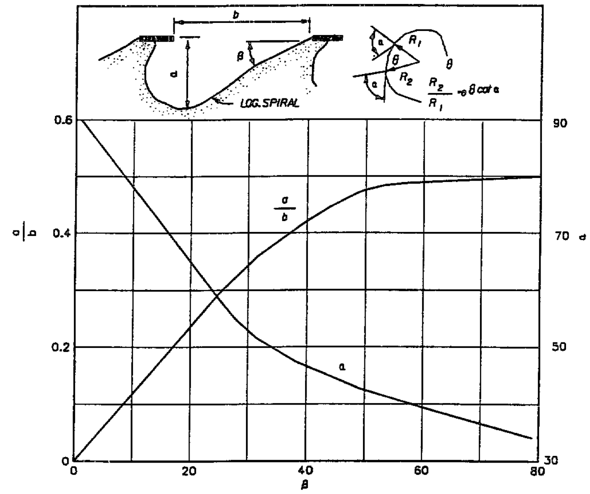


Figure 4-9. Parameters relating to bays in static equilibrium (after Silvester et al. 1980)

The concept has been investigated in the laboratory (Chatham 1972; Sorensen and Beil 1988) and in the field (Inman and Frautschy 1966; Sivard 1971; Douglass and Weggel 1987). Inman and Frautschy (1966) discuss a natural "perched beach" at Algodones in the Gulf of California; Sivard (1971) discusses a man-made perched beach at Singer Island, Florida, constructed of large, sand-filled bags. Douglass and Weggel (1987) discuss the performance of the perched beach at Slaughter Beach, Delaware, built under the Shoreline Erosion Control Demonstration Act of 1974. The sill structure used to construct a perched beach can be considered a special case of a nearshore breakwater, one with a low-crest, a high wave transmission coefficient and extending a relatively long distance along the coast. Whereas the objective of the nearshore breakwater is to shelter a section of the coast from wave action, the objective of the perched beach sill is to introduce a discontinuity into the beach profile. The profile on the landward side is at a higher elevation than the profile on the seaward side.

(2) A dimensional analysis for the design of submerged sills is located in Appendix D. As more experience with perched beaches accumulates, the preceding dimensionless terms can be used to relate the behavior of various installations to each other. Unfortunately, there is currently little experience on which to base a design.